Back-calculated creep rates from case records

Les vitesses de fluage calculées à postériori à partir d'enregistrement de cas

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SYNOPSIS: A number of available settlement observations over a period of 5 to 60 years have been backanalysed to identify the character of the observed deformation rates. From these observed settlement rates it was possible to distinguish between pure creep and primary consolidation.

Generally speaking it was found that the primary in situ process was shorter than the classical one-dimensional consolidation should indicate, and that apparent creep dominated from an early stage. Moreover, it was found that the back-calculated, full-scale in situ creep parameters were comparable to the values usually found in laboratory tests on similar soils and on soft sedimentary rocks. Five subsidence records from various part of the world indicate these same orders of magnitude when interpreted as a lumped creep process.

1 SETTLEMENT POTENTIAL

Sixteen onshore settlement records (δ versus t) are the main basis for the theoretical concept behind this paper. The details of these records were first assembled by Funderud (1986).

From the observed rate of settlement, $\dot{\delta}$, as function of time, t, the settlement potential, S, was defined as the product

$$S = \dot{\delta} t \qquad (1)$$

The 16 field values of S will be compared to the corresponding theories of primary and secondary consolidation (creep).

1.1 Primary potential

When a saturated soil layer of thickness H carries a uniformly distributed load q the soil surface settles initially with a rate $\dot{\delta} = d\delta/dt$ equals

$$\dot{\delta} = \frac{q}{M}\sqrt{\frac{c_v}{\pi t}} \qquad (2)$$

where M = one-dimensional modulus of drained compression (MPa), c_v = coefficient of consolidation (m²/yr) and t = time after load application. Note that $\dot{\delta}$ is independent of layer thickness, H, during the early phase.

The validity of the above formula is limited to 50% degree of consolidation, ie. up to $t \leq t_{50}$, where

$$t_{50} \equiv 0.2\ t_0 \qquad (3)$$

when $t_0 = H^2/c_v$, assuming vertical drainage to the surface level. For a double-drained layer $t_{50} = 0.05\ t_0$.

For $t < t_{50}$ one gets, from Eqs.(2) and (1),

$$S = \frac{q}{M}\sqrt{\frac{c_v t}{\pi}} = \frac{1}{2}\ \delta \qquad (4)$$

The classical theory leads to the degree of consolidation $U = \delta/\delta_p$ as a function of the time factor $T = t/t_0$. Hence, the dimensionless primary settlement potential $p = (dU/dT)T = \dot{U}T$ can be obtained from available U-T curves, leading to

$$S = p\delta_p \qquad (5)$$

where δ_p = 100% primary consolidation settlement.

Fig.1 contains the theoretical value of p as function of T from T = 0 to T ~ 2.5. The potential is parabolic up to p = 0.25 for $T_{50} = 0.2$, and it reaches a maximum value of p = 0.31 for T = 0.4 and then drops asymptotically to 0 past T = 2.0. At the end of the consolidation process it is assumed that primary strain is constant over the whole layer thickness.

1.2 Creep potential

For times $t > t_0$ the excess pore pressure is almost 90% dissipated, in primary consolidation. Hence, both total and effective stress levels remain approximately constant. Nevertheless, several decades of testing and field observations show that settlements are still going on long there after. By classical concepts in rheology this phenomenon is creep (also called secondary consolidation, by a misleading term).

The rate of pure creep is in most saturated soils inversely proportional to time, for $t > t_0$. For an ideal layer of thickness H, laboratory and field experience over a 20 year period have shown that

$$\dot{\delta} \equiv \frac{H}{r_s t} \qquad (6)$$

where r_s = creep resistance (or time resistance, Janbu 1969).

Hence, the creep potential $\dot{\delta}$ t becomes constant,

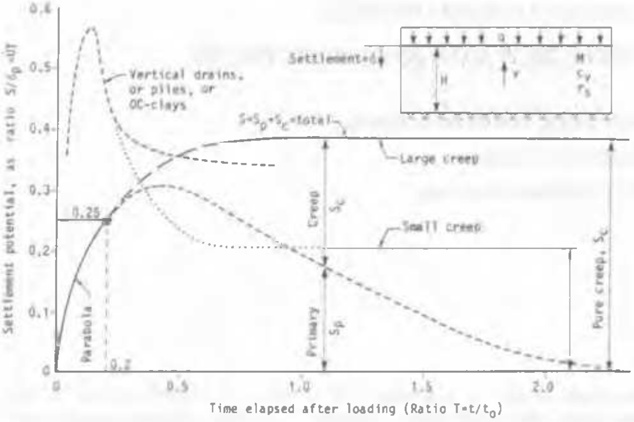

Figure 1. Settlement potential versus time, dimensionless scales.

$$S = \frac{H}{r_s} \tag{7}$$

From the experimental background, the following orders of magnitude may be cited for r_s:

Peat, organic clays	r_s = 10 - 100
Clays and silts	r_s = 100 - 300
Silty sand to medium sand	r_s = 300 - 1000
Stiff clays, dense sand	r_s =1000 - 2000
Sedimentary rocks	r_s = 200 -10000+

The lowest creep resistances in sedimentary rocks correspond to loose particle bonding (underconsolidated state), or to large stress changes causing partial collapse of particle bonds or cementations.

The back-calculated in situ data will be compared to the gross values obtained mainly from tests on 2 cm samples, while the sediment thicknesses for the on-shore case records are several tens of meters, and kilometers off-shore.

1.3 Total potential, S

In reality, primary consolidation and creep are overlapping over a long time-span. Since total settlement is the sum of the two, $\delta = \delta_p + \delta_c$, the total potential at any time is equal to

$$S = S_p + S_c \tag{8}$$

where S_p should dominate in the beginning ($t < 0.2t_0$) while S_c should dominate late in the process ($t > t_0$). This is illustrated in Fig.1.

For $T < 0.2$ the primary may dominate completely, while S may be almost constant for $T > 0.5$ (or earlier). From $T \sim 0.5$ to $T \sim 2.0$ primary and creep are linked. For cases of large creep the contribution from creep and primary consolidation may be equal at $T - 1.0$. However, from a modeling point of view the total settlement in such cases may be adequately described by the equivalent constant creep potential already from $T = 0.5$ or even earlier. Vertical sand drains, and/or piles give a sharp rise to higher values of S during the early phase of the settlement, then S drops drastically down to S_c even in cases of "small creep", Fig.1. This same tendency may also show up in the overconsolidated stress ranges.

2 EXAMPLES OF CASE RECORDS

Out of the sixteen cases, two will be chosen as examples of application of the settlement potential concept.

The Steinkjer grain silo is founded on 26 m long precast concrete piles, driven through layers of organic fill and sandy layers, into silty fine sand, overlying layered overconsolidated clay to bedrock at roughly 70 m depth. The maximum excess load under the concrete floor is about 300 kPa, but the load varies with the grain pay-load. The settlement prognosis was 30 cm + 5 to 10 cm for load fluctuations over some 10 to 20 years.

The simplified profile, and the observed average settlement from 4 observation points are shown in Fig.2. The rate of settlement $\dot{\delta}$ was obtained as the tangent to the (δ-t) curve at several times t. The product $S = \dot{\delta}t$ was established, and it is plotted versus time in Fig.2.

The maximum value of S is 9 cm after 1 year or so. Then S drops to about 4 cm after 3 years, and remain almost constant thereafter. At one of the observation points S dropped to about 2 cm, but appeared to increase slightly from then on. The piles seem to have the same effect as vertical sanddrains, see Fornebu Airport values in Fig.4

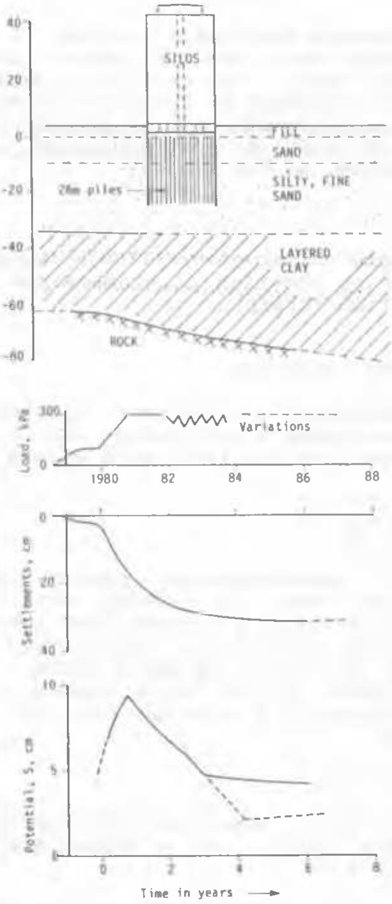

Figure 2. Steinkjer grain silo.

The time of observation at the grain silo is too short for precise evaluation of the creep potential, but in average it is seen to be about $S_c \sim 3.5$ cm. Using the soil thickness below the piles as H = 40 m one gets an average $r_s = H/S_c = 1150$, which is in fair agreement with ordinary laboratory values for such soil deposits.

The Railroad Customs Building in Oslo were founded on very short timber piles below the raft. It was built in the period around 1920 to 25. The soil profile is given in very broad terms in Fig.3. The lower 20 to 30 m is indicated as sandy, gravelly (possibly moraine). The upper 35 m has been thoroughly investigated (Andersen, 1968). It is broadly speaking a stratified, medium stiff clay.

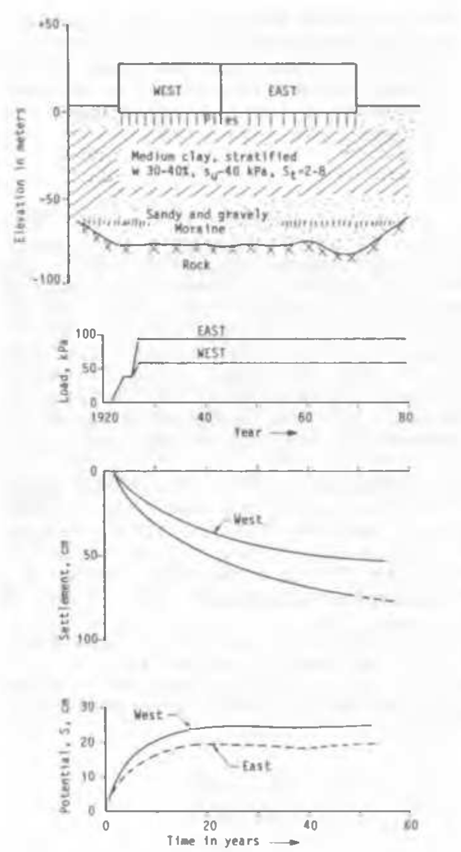

Figure 3. Railroad Customs, Oslo

The net loads on the east and the west part of the building is shown in Fig.3, and the average settlement observations for these two parts are also given as function of time.

The settlement potential, S = δt, is seen to be almost as in ideal theory. The first part is of a parabolic form, whereafter it becomes constant and equal to S_c = 24 cm, after 10 to 15 years, and it remains so for another 30-35 years.

Assuming the thickness of the clay layers to be H = 50 m one would find an average creep resistance in the field to be $r_s = 5000/24 = 210$ which again agrees with laboratory data obtained from 2 cm thick samples.

3 CASE RECORD SUMMARY

The backcalculated settlement potentials for all 16 cases are assembled in 3 diagrams in Fig.4.

For the structures on sand-silt deposits in diagram (a), the settlement potential is almost constant, or slightly decreasing after just 2 years or so. For the next 15 years a creep potential of S_c = 2-5 cm seems to cover all the field records. The total thickness H of the layered deposits varies between 25 to 45 m, with 35 m as an average. Hence the in situ creep resistance $r_s = H/S_c$ lies in the range of 700 to 1800, which covers the creep resistances obtained by laboratory testing quite well.

For the Statfjord A gravity platform (Fig.4b), the "primary process" terminated after just one year. Since then the creep potential S_c has been fairly constant and equal to 4.5 cm in average. The equivalent, average thickness of influence is difficult to judge, but from the soil profile one may assume H = 60-75 m, in which case r_s = 1350 to 1700. This interval is in fair agreement with laboratory values for such stiff overconsolidated clays as on Statfjord. The observation years of the B-platform are too few to go into details, but apparently it is a somewhat stiffer subsoil.

Diagram (c) contains the settlement potentials for foundations on soft to medium stiff clays. The 10 m high Fornebu Airport embankments were placed on a very soft, organic top clay layer underlain by a soft to medium stiff clay. The soil thickness varied between 10 to 20 m approximately. Sanddrains were installed to 12 m depth.

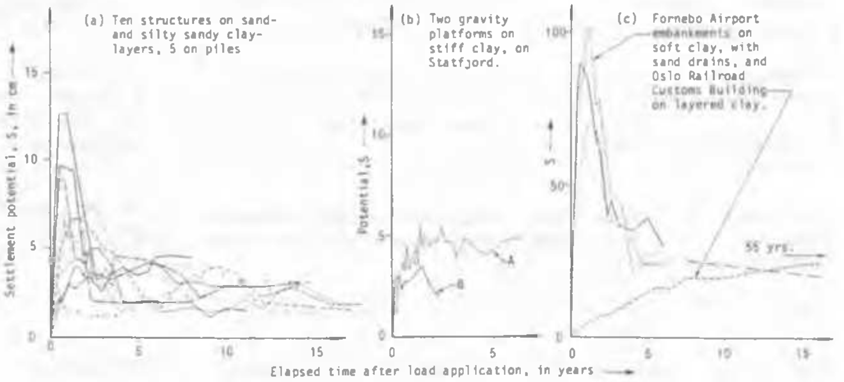

Figure 4. In situ settlement potentials from 16 cases.

Based on the average settlement records for three sections, the settlement potential was calculated as shown in Fig.4, diagram (c). The maximum S is 70 to 110 cm after roughly 1 year, then it drops quickly to 35 to 45 cm after 2 years, and reaches an almost constant value of 30 cm after 5 years.

Assuming H = 15 m as average thickness, and S_c S_c = 30 cm one gets $r_s = 1500/30 = 50$. This low creep resistance is due to the organic content of the soft clay. With time, scarce observations indicate that S_c drops, which means that the creep resistance r_s increases. This phenomenon is also observed in laboratories, quite frequently.

The lower curve in diagram (c) in Fig.4 belongs to the Railroad Custom Building previously dealt with in Fig.3. Only the first part of the curve is shown, up to 16 years. For the remaining 40 years the creep potential is constant, S_c= 24 cm, corresponding to r_s = 210, see Fig.3.

4 SOME SUBSIDENCE RECORDS

A number of subsidence records are available in the form of observed rates of terrain subsidence, $\dot{\delta}$. Even though the whole record is seldom available, the maximum rate $\dot{\delta}$ at a certain time t may often be known. If so, it is a simple matter to obtain the potential $S = \dot{\delta}t$ at that time. Five such records are given in gross numbers in Table 1. The values of $\dot{\delta}$ corresponds to t = 10-12 years or 12-15 years after the start of oil, gas or water production. The total thickness of the deposits involved is also indicated in Table 1.

Table 1. Creep resistance from subsidence

Case	$\dot{\delta}$	S	H	$\bar{r}_{se}$
Wilmington	0.60	6 - 7	1000±	140-165
Ekofisk	0.30	3 - 3.5	3000	800-1000
Wairakei	0.25	2.5-3.0	450±	150-180
Mexico City	0.30	3.5-4.5	750±	165-210
Bangkok	0.10	1 - 1.2	200±	170-200
Dimension	m/yr	m	m	1

In all 5 cases the sediments are strongly layered. In theory the creep potential for such layered systems is

$$S_c = \Sigma \frac{\Delta H}{r_s} = \frac{H}{\bar{r}_{se}} \qquad (9)$$

where $\bar{r}_{se}$ = equivalent, average creep resistance.

The calculated value of $\bar{r}_{se}$ in Table 1, is an overall average value. However, it could also have been obtained from known r_s-values in each layer ΔH by the formula

$$\bar{r}_{se} = \frac{H}{\Sigma \frac{\Delta H}{r_s}} \qquad (10)$$

It is surprising to see that four of the cases lead to creep resistances between 150 to 200, corresponding to ordinary values for clay. An exception is Ekofisk, leading to an average creep resistance so typical for sand, namely 800-1000. However, the reservoir and the overburden of sedimentary rock at Ekofisk contain layers of widely different $\bar{r}_s$-values, perhaps within the whole range of 200 to several thousand.

Once S is known (or assumed) the rate of settlement is obtained from Eq.(1)

$$\dot{\delta} = \frac{S}{t} \qquad (11)$$

Hence, the settlement itself, assuming t_i for load application, when $\delta = \delta_i$,

$$\delta = \int_{t_i}^{t} \frac{S}{t}\,dt + \delta_i \qquad (12)$$

If creep dominates from $t \geq t_c$ with a creep potential S_c = constant, Eq.(12) lead to

$$\delta_c = S_c \ln \frac{t}{t_c} \qquad (13)$$

Example: S_c = 10 cm, t_c = 1 year, then the following 9 years will lead to $\Delta\delta = 10$ cm $\ln 10$ = 23 cm. The next 10 years add $\Delta\delta = 10$ cm $\ln 2$ = 7 cm, while another 10 years lead to less than 4 cm increase. Hence, after 30 years $\delta = 34$ cm + δ_i.

5 SHORT RESYME

The settlement potential concept $S = \dot{\delta}t$ has been used in back-analysing 16 onshore settlement records, and 5 subsidence observations. The longest onshore records of S clearly indicate an almost constant value after very few years, corresponding to a pure creep potential = S_c.

Converting the in situ creep potentials from deep deposits into dimensionless creep resistances, one finds that the average field values are in fair agreement with the creep resistances r_s obtained by laboratory testing of samples just a few cm thick.

As an example, consider a H = 30 m thick deposit of layers of clay, silt and sand corresponding to r_s-values between 150 and 1500, leading to creep potentials $S_c = H/r_s$ = 2 cm to 20 cm. A quick look at Fig.4 will show that there is a lot of field evidence to support this order of magnitude.

Because of the simplicity and the versatility of the concept, the strain potential, $\dot{\varepsilon}t$, is being implemented in our laboratory as a standard routine of oedometer test interpretation, for each load step in IL-tests.

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